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Modelling progressive failure of steel moment frames exposed to localised fire

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Abstract. This paper presents modelling of the progressive failure of steel moment frames subjected to localised fire. A finite element approach using the software ABAQUS has been developed to analyse the structure. An explicit-dynamic solution was adopted to solve the non-convergence problems caused by element buckling. A series of validation analyses were carried out to ensure that the results were within an acceptable level of accuracy. The analysis results are shown to match well with the previous experimental data and analysis. This modelling approach allows detailed insights to be obtained into the structural robustness of such frames in fire situations.

Keywords: building collapse, finite elements, fire engineering, robustness, steel frame

1. Introduction

Progressive collapse is typically triggered by the spread of initial local failure from element to element subsequently leading to the collapse of a whole structure or a large portion of it. In an accidental situation such as fire, structural engineers need to ensure that buildings maintain their stability to prevent such collapse. The most notable case of this type of failure is the collapse of the World Trade Centre towers in 2001 due to a large uncontrolled fire. Given numerous fatalities and losses in past events, it is essential to study the collapse resistance of buildings under fire conditions.

Conventional fire design is based on standard fire tests on isolated individual elements[1]. This approach does not represent realistic behaviour of the structure. The boundary conditions and member interactions within a structural system may affect the behaviour of the element. The UK Cardington tests[2–4] indicated that a composite steel frame had better fire resistance compared to those of single elements in the standard fire test. This shows the importance of investigating the complete structure to understand the behaviour of the building under fire conditions. However, full-scale experiments of the global behaviour of the structure in fire conditions are very expensive and extremely time-consuming. Thus, it seems that conducting a large number of such experiments is not feasible in the study the progressive collapse of buildings in a fire.

A numerical analysis using finite elements is an alternative way to study structural behaviour in fire. In a structural fire analysis, the material behaviour is more complicated than at ambient temperature. The materials become weaker and more flexible at high temperature. Moreover,



geometric and material nonlinearities make the analysis potentially complicated. Thus, a robust numerical model is required to analyse the behaviour of the building under fire conditions.

This study presents a modelling approach to simulate the progressive failure of a steel frame building exposed to localised fire. The study aims to examine the behaviour of a realistic structure during the progressive collapse process. The model adopted in this study is validated against experimental results and previous analysis.

2. Description of frame tests

A steel frame, which has been previously tested by Jiang[5], and subsequently analysed by Jiang et al.[6] using a different finite element approach, was selected in this study. Both the experiment and analysis provide reliable data on the behaviour of the heated column, including post-buckling and the effect of the cooling phase on the progressive collapse analysis.

Figure 1 shows the test frame. All steel columns and beams were rectangular hollow structural sections. The columns had a section of 50 x 30 x 3 mm, with a yield strength of 380 MPa and Young's modulus of 200 GPa at ambient temperature. The column bases were fixed at the ground, and out-of-plane displacements at the middle of the beams were restrained. The beams had a section of 60 x 40 x 3 mm with a yield strength of 306 MPa and Young's modulus of 200 GPa. For practical application, the thermal expansion of steel was taken as $1.4 \times 10^{-5} \text{ }^{\circ}\text{C}^{-1}$ [1].

The frame was initially loaded by gravity load. Then, column C3 was heated together with the right and the left side of the beams, as indicated in Figure 1. The loading temperature consists of heating the test frame to 800 °C, then cooling it to ambient temperature, as shown in Figure 2.

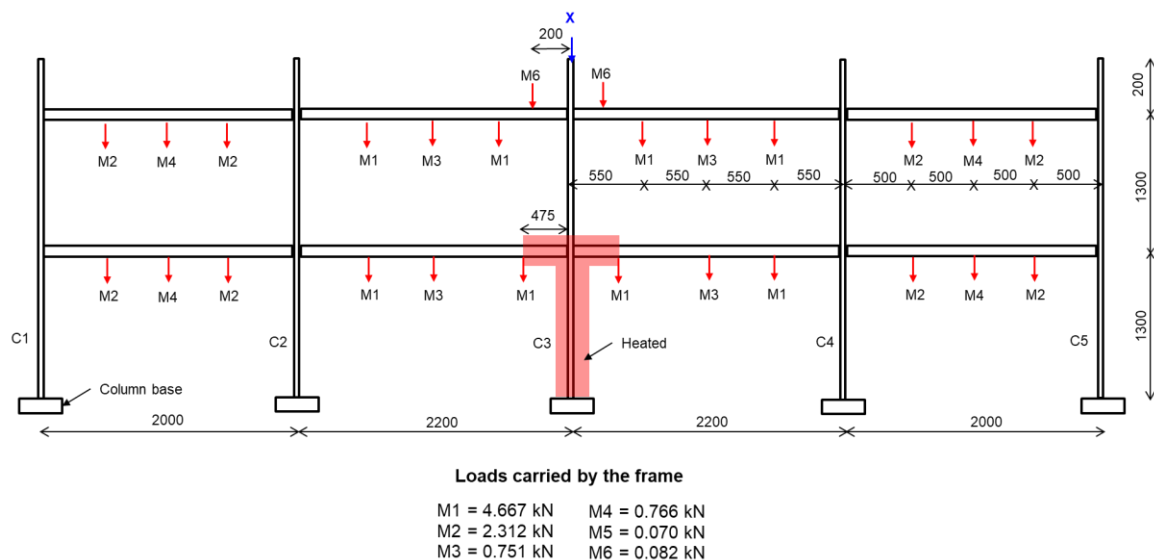


Figure 1: Test frame schematic [5]

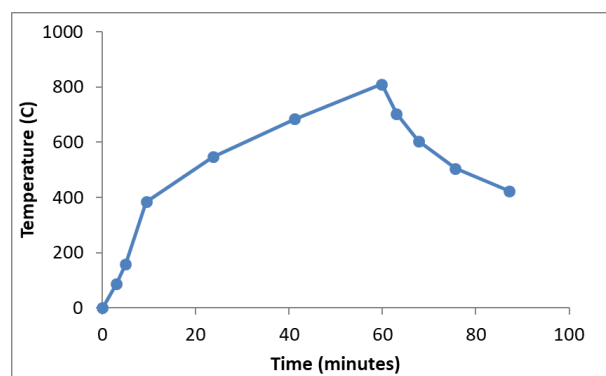


Figure 2: The temperature loading

3. Numerical model

The finite element software ABAQUS v6.13[7] is utilised to model and analyse the steel frame. As shown in Figure 3, beams and columns are discretised using 1-D line elements. It is worth noting that localised buckling cannot be captured using the 1-D line elements. The connections are assumed to be rigid for simplicity, thus, connection failure is not taken into account in the present study.

Elastic-perfectly plastic structural behaviour is assumed for the steel elements and temperature-dependent properties according to Eurocode 3[8] are adopted. The steel temperature within the cross-sections is considered to be uniform as the steel section used in this study is sufficiently thin. The structural temperature can be directly applied as an amplitude load to the elements.

An explicit dynamic solver is employed to solve numerical convergence difficulties[6,9–11]. This analysis also considers structural dynamic effects. Rayleigh damping is used with 5% viscous damping, and the Johnson-Cook model [12] is adopted to model possible strain rate effects in the steel. The real time is scaled down to reduce the computational time of analysis.

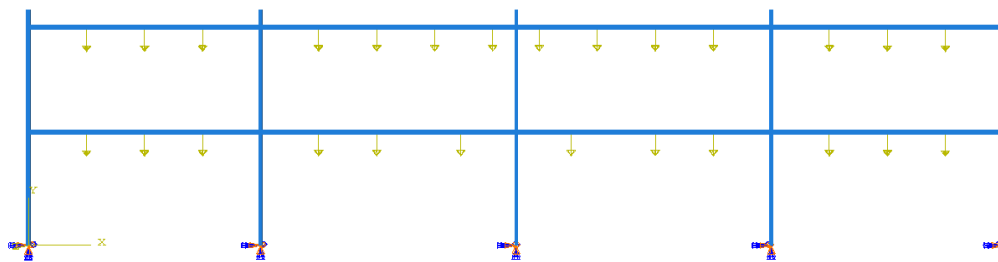


Figure 3: ABAQUS model

4. Progressive collapse analysis

Figure 4 shows the axial displacement at the top of column X (indicated in Figure 1) against temperature with different running time compared to the experimental data and the previous analysis by [6]. In general, the analysis agrees with the previous studies. A running time of 9 s matches well with the experiment and the previous analysis, with 0.5% error, as shown in Table 1. It can be seen that a scaled analysis time can cause oscillation of response. Increasing the time step results in a reduction in the oscillation, but the computational efforts will increase significantly. Figure 5 shows the deformed shape of the steel frame.

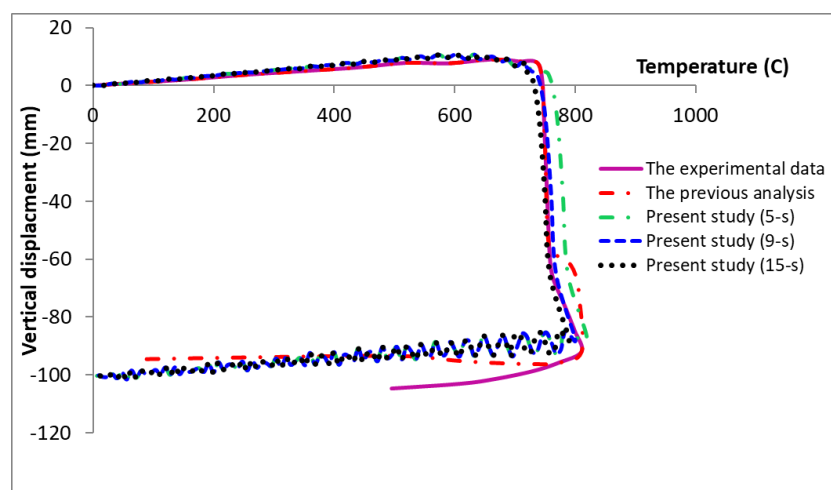


Figure 4: The axial displacement at the top of column X

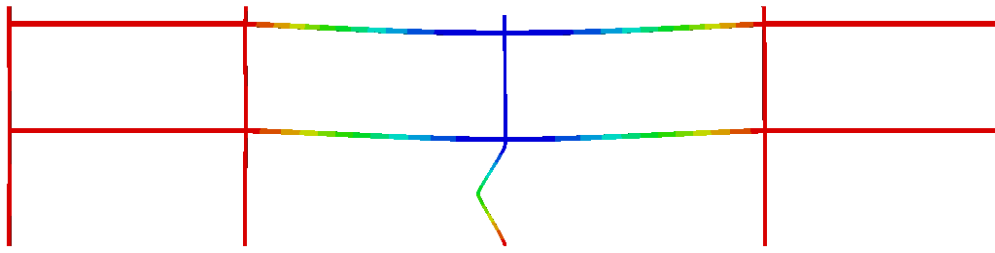


Figure 5: Deformation of the steel frame

Table 1: Buckling temperature and the percentage of error

<i>Time step</i>	<i>Buckling temperature (°C)</i>	<i>Error percentage (%)</i>
Test (90 minutes)	749	0
5	749	2.00
9	745	0.53
15	740	1.2

The variation of axial force against temperature in column C3 (heated) and column C2 (cool) are presented in Figure 6. Initially, the axial force at the heated column (C3) increases due to thermal expansion of the column. This causes the reduction of the axial force at the adjacent column (C2). Once the temperature reaches 730°C, buckling occurs in the heated column since the axial force reaches its buckling resistance. At this stage, the heated column loses load-bearing capacity, and the axial force suddenly reduces. Then, the loads previously sustained by the heated column are transferred to the adjacent column (C2). Thus, the axial force of the adjacent column increases.

During the cooling stage, the heated column contracts downward instead of returning to the initial position, as shown in Figure 3. In this case, tension occurs in the heated column and the additional loads are shifted to the adjacent column. This is confirmed by the variation of the axial forces, as shown in Figure 6. This indicates that the cooling stage can be more dangerous than the heating stage.

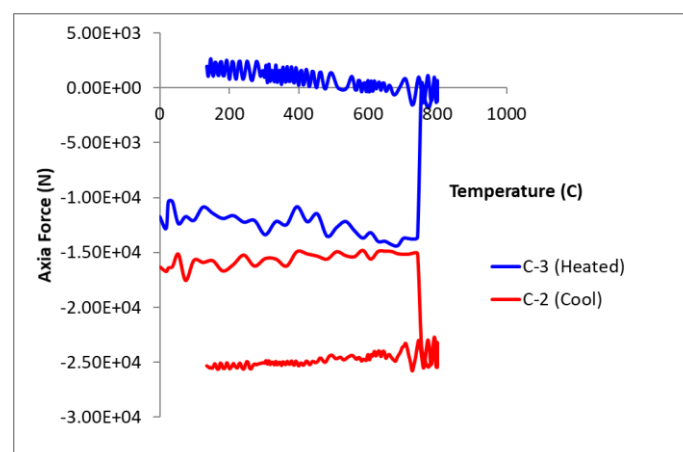


Figure 6: The axial force of the heated column (C3) and the adjacent column (C2)

The analysis above shows the load redistribution path and member interaction within the steel frame. Although significant deflection occurs, there is no collapse of the steel frame for this scenario. This is because the loads previously resisted by the heated column are safely transferred to the

adjacent column. This demonstrates that the failure of an individual element may be acceptable when the total collapse can be prevented.

5. Conclusions

Based on this study, the following key conclusions are noted:

1. The results of the analysis agreed well with the experimental data and previous analysis so that the modelling approach is suitable to simulate the progressive collapse of a steel frame.
2. It is feasible to adopt an explicit dynamic solver for analysis of structures in a fire. The real running time of analysis can be scaled down to save computational cost.
3. This analysis shows the importance of the cooling phase in the progressive collapse analysis. The results reveal that additional loads are transferred from the heated column to the adjacent column during the cooling phase.
4. The analysis captures the load redistribution path when the failure of an individual element occurs. Due to load redistribution, the failure of a single structural component may still be permissible when the total collapse of the structure can be prevented. Consequently it is essential to investigate the complete structure to understand the actual behaviour of the building in a fire.

6. Limitations and recommendations for future studies

This study has achieved its objective to simulate the behaviour of a realistic structure during a progressive collapse. However, it is believed that further studies are required. Different fire scenarios and structural arrangements need to be studied to investigate their effect on the collapse resistance of the steel moment frame.

Furthermore, the 1-D line element used in this study cannot capture all possible failure modes, such as local buckling. An experiment study by Wang and Li[13] showed that there is a possibility that the failure of the column can occur prematurely due to localised buckling. In this case, 2D or 3D elements are required to capture the localised buckling.

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